Chapter 7
Examples Demonstrating Methodology

7-1. Introduction

a. Two littoral budget analyses are presented in this Chapter. The first is an east coast example for a stretch of coast near Asbury Park, New Jersey. This analysis is a condensation of the budget developed by Gravens, Scheffner, and Hubertz (1989). The motivation for this littoral budget was to examine the shadowing effect of Long Island on the local wave climate of the study reach. The second example is for Oceanside, California, and is a condensation of the sediment budget analysis conducted by Kraus et al. In preparation, also discussed by Simpson, Kadib, and Kraus (1991). This analysis was developed to determine the impacts of various management alternatives.

b. The level of detail is quite different for the two littoral budgets presented. The appropriate level of analysis is a function of the intended uses of the littoral budget, and the actual level of analysis may depend also on the available resources to complete the project. However, both include the essential components of a budget analysis. These begin with a site description, background, and examination of previous analyses. An examination of past and present conditions and the results of other studies should be completed before initiating a new budget analysis.

c. Next is the determination of the longshore sediment transport rate. This requires data on wave conditions over as long a time period as possible. These waves are propagated to and transformed in the surf zone. Appropriate sediment transport equations must be selected and incorporated in a shoreline change model. This model must be calibrated and sensitivities to boundary conditions examined. Ideally, historical shoreline positions and wave conditions are available for the same period of time to allow this calibration.

d. The actual determination of the budget can then be completed. Usually there are poorly quantified components remaining in the analysis, such as offshore gains and losses. These must be estimated using any available data, engineering judgment, and the requirement that the budget close. Although a significant effort goes into the development of a littoral budget, it must be remembered that it is an estimate and can easily be in error by a factor of two. The budget is calibrated with shoreline positions over a number of years and indicates long-term average rates of change. It may not be indicative of the changes in any one year.

7-2. East Coast Example: Asbury Park to Manasquan Inlet, New Jersey

The sediment budget of the New Jersey coast from Asbury Park south to Manasquan Inlet was studied by Gravens, Scheffner, and Hubertz (1989) and is discussed here because several features of that study provide guidance for similar calculations at other localities. To solve the budget, the authors employed WIS information to calculate sediment transport at specific locations, shoreline movement information from photo and map analyses in determining sand volume changes, knowledge of processes important to the area based on previous studies, and engineering judgment regarding adjustments to transport rates.

a. Site description. The study area is between Asbury Park and Manasquan, New Jersey. It is a sandy stretch of coastline 8.5 miles long (Figure 7-1) with 25- to 150-ft-wide beaches. This reach has 81 groins, two structurally stabilized tidal inlets, and intermittent sections of sheet pile and wood bulkheads. There is no coastal dune in the study area.

b. Background. Correct nearshore wave data are essential for calculating sand transport rates. These transport rates are used to estimate or verify shoreline changes. The purpose of the sediment budget analysis by Gravens, Scheffner, and Hubertz (1989) was to confirm that wave shadowing by Long Island to the north was properly represented in the wave hindcast time series for subsequent shoreline change simulation. Wave shadowing was confirmed by the agreement of historical transport rates with those calculated using WIS data and the energy flux method.

c. Previous analyses. The historical average transport rates were calculated from differences in survey measurements made from Mantoloking (approximately 5 miles south of Manasquan Inlet) northward to Sandy Hook, New Jersey, at various times from 1838 to 1935, and reported by Caldwell (1966). Sandy Hook accumulated sand at the northern boundary at the average rate of 493,000 cu yd/year for that period, and the loss of sand from that location was considered zero. Thus the northward transport rate at Sandy Hook was established at 493,000 cu yd/year. Caldwell estimated that the northward transport was uniform along the study area, based on an earlier study that showed wave energy approaching from the southeast quadrant was uniform

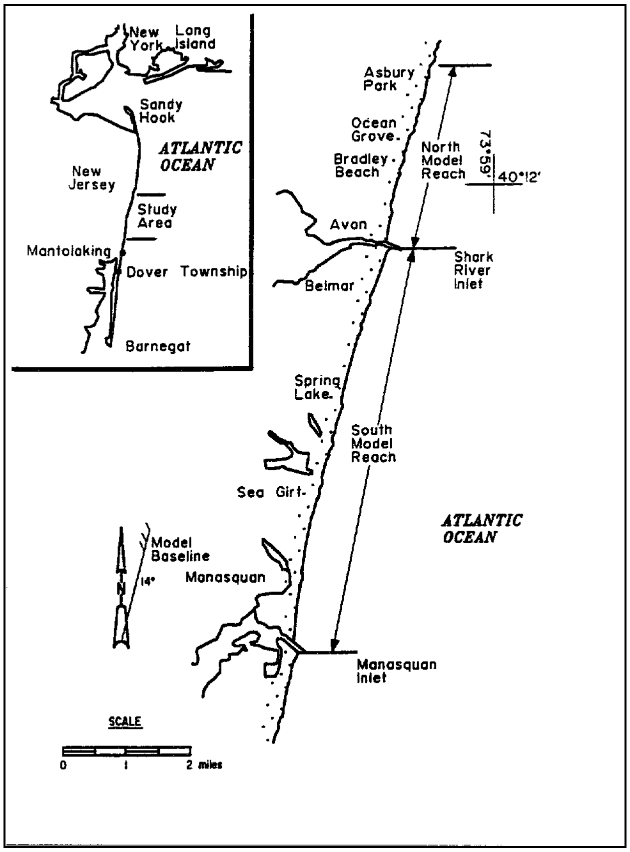

Figure 7-1. Location of Asbury Park to Manasquan study area

over the New Jersey coast. However, wave energy from the northeast quadrant is diminished at Sandy Hook because of sheltering by Long Island, but farther south along the coast there is no sheltering effect. The result of these patterns is a nodal point of net littoral transport at Dover Township, 35 miles south of Sandy Hook. Evidence showed the 493,000 cu yd/year accumulation at Sandy Hook came from the beach face in that 35-mile distance. Measured erosion averaged 723,000 cu yd/year, but about one-third of that volume was material finer than sand and, once eroded, was lost from the system.

(1) The study area was divided into four segments based on locations where transport rates were known or could be inferred: (a) Mantoloking to Manasquan, (b) Manasquan to Asbury Park, (c) Asbury Park to base of Sandy Hook, and (d) base of Sandy Hook to tip of Sandy Hook. At the south end of the study area, Caldwell reasoned that net transport into the segment Mantoloking to Manasquan was zero, but measured differences in surveys indicated net transport out of the reach to be 74,000 cu yd/year. With the sand transport rates established at the two ends of the study area and sand volume changes measured for all the reaches, the sediment budget could then be solved, as illustrated in Figure 7-2. In this figure the control volume represents a reach of the littoral stream, and a quantity eroded from the shore face is considered as flux into the control volume.

d. Wave conditions. Wave summary statistics are available from WIS hindcasts at five stations located in 33-ft water depth along the study reach (Figure 7-3). Percent occurrence statistics are tabulated by period band and height increment for angle bands of 30°. Table 7-1 lists statistics for Station 54, located near Sandy Hook, as an example. The angles reported for these WIS Phase III stations are oriented to the shoreline trend and are measured counter-clockwise from the shoreline. Transport calculations required that central angles of the given angle bands be referenced to shore-normal. By convention, waves approaching shore at positive angles cause transport to the right when looking seaward from shore, and waves of negative angles cause leftward transport.

e. Description of Transport Algorithm. Wave input was developed for a program to calculate wave transformation by linear wave theory and assumption of straight and parallel contours and longshore sand transport by the CERC formula. The CERC formula is based on the energy flux method, an empirical correlation between transport rate and the longshore component of wave power evaluated at the breaker zone (Equations 6-1 and 6-2). A program to calculate wave transformation to breaking and resulting sand transport was developed by Gravens (1988, 1989).

f. Model calibration. The input and calculated results are listed in Table 7-2. To investigate alongshore variation in transport, average height and weighted average period were calculated for each angle band at each hindcast station. Transport rates calculated for reaches corresponding to the wave hindcast stations were compared with those determined from survey measurements reported by Caldwell.

(1) This first effort at using WIS summary statistics yielded an alongshore trend in transport rates which agreed with that of the Caldwell study, but magnitudes differed greatly. Upon closer examination of representative shoreline orientations for each calculation reach, small differences were found between angles used in the Phase III transformations and the shoreline angles measured from a small-scale map. The decision was made

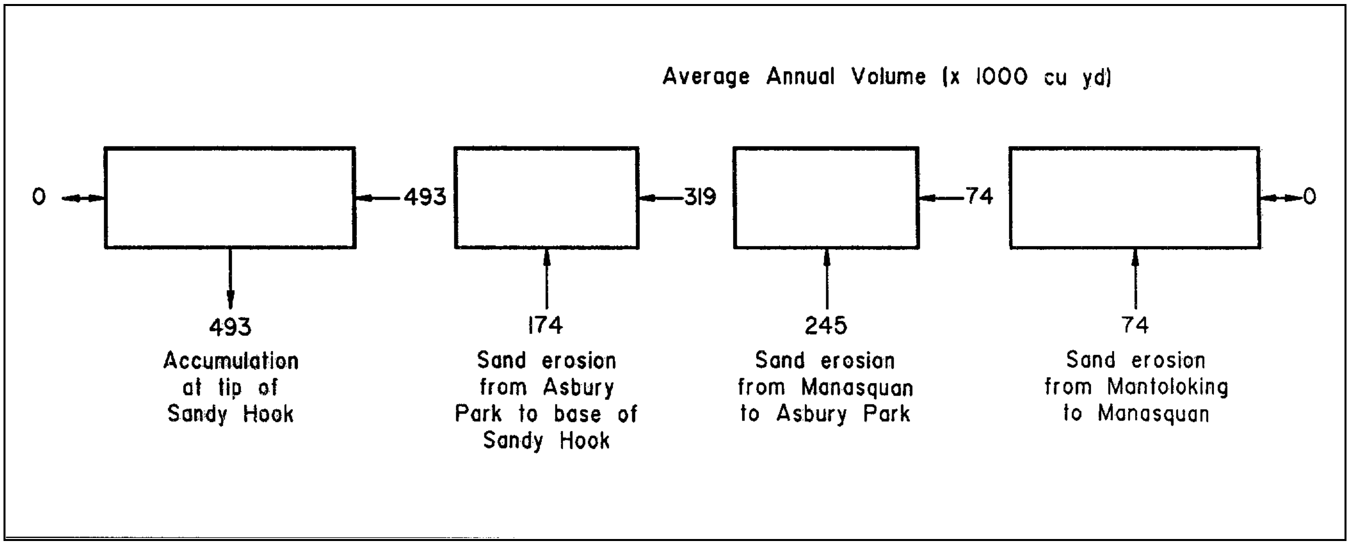

Figure 7-2. Sediment budget from Sandy Hook to Manasquan, New Jersey, 1838 - 1935 (after Caldwell, 1966) (cu yd/year)

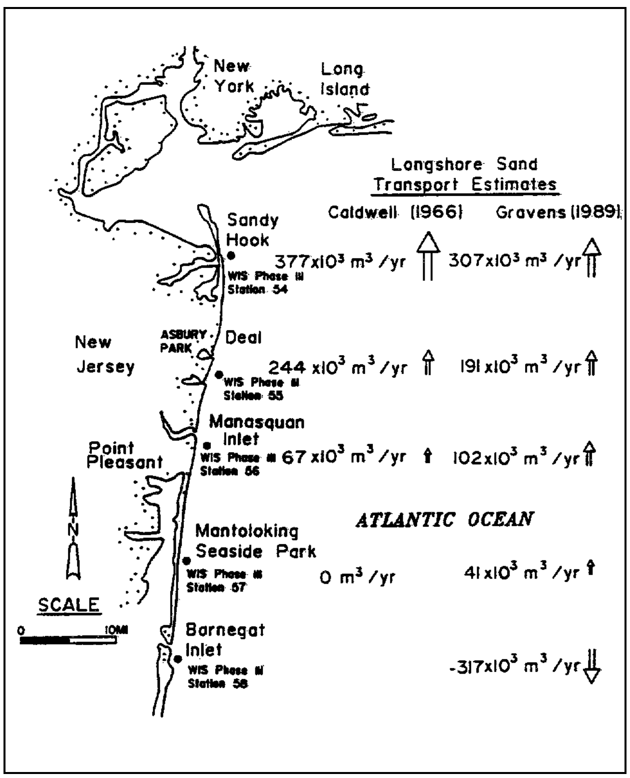

Figure 7-3. Potential longshore sand transport rates (Gravens, Scheffner, and Hubertz 1989)

to apply corrections to the angles in the WIS statistics and recalculate transport rates. Table 7-3 illustrates the difference in transport rates made by a systematic 8-deg change at Station 54. The proper alongshore trend was again reproduced by recalculating transports for the five stations, and results showed an improvement in transport rates with respect to historical averages.

g. Sediment budget. Because of the success of the preliminary calculations, a more refined discretization of the WIS Phase II data was undertaken. Within each angle band, distributions of height and period are tabulated by height categories of 0.5-m increments and period categories of 1 sec. The median height and the weighted average period within each height category were then input to the longshore transport routine. Twenty-eight height-period-direction combinations were required to represent the wave climate effective in transporting sand alongshore (Table 7-4). Calculated left and right transport rates for the reaches yield net rates that correspond well with average historical rates, and both are shown on Figure 7-3. This analysis established confidence that the WIS time series could be utilized as input to the shoreline change model, GENESIS, and would permit accurate shoreline change simulation. The sediment budget studies showed that shadowing of the waves by the large land mass to the north was accounted for properly.

Table 7-1
Percent Occurrence by Period and Angle Band for WIS Phase III Station 54

Station 54 20 Years Wave Approach Angle (Degrees) = 0.0-29.9
Shoreline Angle* = 4.0 Degrees Azimuth
Water Depth = 10.00 Meters
Percent Occurrence (X1000) of Height and Period by Direction

Height (Meters)	Period (Seconds) 0.0-2.9	3.0-3.9	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-Longer	Total
0.00 - 0.49	2888	5985	4065	2347	588	869	313	11	6	.	17072
0.50 - 0.99	.	313	1976	1213	388	179	47	5	6	1	4128
1.00 - 1.49	.	.	17	77	100	6	.	.	1	.	201
1.50 - 1.99	.	.	.	.	10	1	.	.	.	.	11
2.00 - 2.49	.	.	.	.	.	.	.	.	.	.	0
2.50 - 2.99	.	.	.	.	.	.	.	.	.	.	0
3.00 - 3.49	.	.	.	.	.	.	.	.	.	.	0
3.50 - 3.99	.	.	.	.	.	.	.	.	.	.	0
4.00 - 4.49	.	.	.	.	.	.	.	.	.	.	0
4.50 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - Greater	.	.	.	.	.	.	.	.	.	.	0
Total	2888	6298	6058	3637	1086	1055	360	16	13	1	

Average HS(M) = 0.32 Largest HS(M) = 1.85 Angle Class % = 21.4

Station 54 20 Years Wave Approach Angle (Degrees) = 30.0-59.9
Shoreline Angle* = 4.0 Degrees Azimuth
Water Depth = 10.00 Meters
Percent Occurrence (X1000) of Height and Period by Direction

0.00 - 0.49	944	1254	.	.	3237	7493	3879	302	494	465	18059
0.50 - 0.99	.	821	1651	340	130	1481	795	80	205	8	5511
1.00 - 1.49	.	.	114	665	316	485	248	53	37	.	1918
1.50 - 1.99	.	.	.	25	154	309	109	20	10	1	628
2.00 - 2.49	.	.	.	.	27	90	51	10	.	.	178
2.50 - 2.99	.	.	.	.	.	3	11	11	1	.	26
3.00 - 3.49	.	.	.	.	.	.	.	3	.	.	3
3.50 - 3.99	.	.	.	.	.	.	.	.	.	.	0
4.00 - 4.49	.	.	.	.	.	.	.	.	.	.	0
4.50 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - Greater	.	.	.	.	.	.	.	.	.	.	0
Total	944	2075	1765	1030	3864	9861	5093	479	747	465	

Average HS(M) = 0.47 Largest HS(M) = 3.05 Angle Class % = 26.3

(Continued)

(Sheet 1 of 3)

Table 7-1
(Continued)

Station 54 20 Years Wave Approach Angle (Degrees) = 60.0-89.9
Shoreline Angle* = 4.0 Degrees Azimuth
Water Depth = 10.00 Meters
Percent Occurrence (X1000) of Height and Period by Direction

	Period (Seconds)										
Height (Meters)	0.0-2.9	3.0-3.9	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-Longer	Total
0.00 - 0.49	634	744	.	.	593	2861	2286	992	278	455	8843
0.50 - 0.99	.	571	1309	148	37	718	631	277	311	106	4108
1.00 - 1.49	.	.	111	693	147	164	131	30	70	58	1404
1.50 - 1.99	.	.	.	34	256	159	71	6	11	25	562
2.00 - 2.49	.	.	.	.	29	285	80	5	1	25	425
2.50 - 2.99	.	.	.	.	.	51	92	5	1	1	150
3.00 - 3.49	.	.	.	.	.	.	6	10	5	.	21
3.50 - 3.99	.	.	.	.	.	.	.	8	.	.	8
4.00 - 4.49	.	.	.	.	.	.	.	.	.	.	0
4.50 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - Greater	.	.	.	.	.	.	.	.	.	.	0
Total	634	1315	1420	875	1062	4238	3297	1333	677	670	

Average HS(M) = 0.61 Largest HS(M) = 3.70 Angle Class % = 15.5

Station 54 20 Years Wave Approach Angle (Degrees) = 90.0-119.9
Shoreline Angle* = 4.0 Degrees Azimuth
Water Depth = 10.00 Meters
Percent Occurrence (X1000) of Height and Period by Direction

0.00 - 0.49	908	1839	1471	1617	1358	1795	961	123	78	432	10582
0.50 - 0.99	.	116	956	1018	236	1651	1247	210	333	1156	6923
1.00 - 1.49	.	.	17	349	316	586	545	102	85	313	2313
1.50 - 1.99	.	.	.	8	123	381	217	44	15	131	919
2.00 - 2.49	.	.	.	.	13	154	138	6	6	23	340
2.50 - 2.99	.	.	.	.	.	15	49	3	1	15	83
3.00 - 3.49	.	.	.	.	.	.	1	1	.	3	5
3.50 - 3.99	.	.	.	.	.	.	.	1	.	1	2
4.00 - 4.49	.	.	.	.	.	.	.	.	.	.	0
4.50 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - Greater	.	.	.	.	.	.	.	.	.	.	0
Total	908	1955	2444	2992	2046	4582	3158	490	518	2074	

Average HS(M) = 0.61 Largest HS(M) = 3.68 Angle Class % = 21.2

(Continued)

(Sheet 2 of 3)

Table 7-1
(Concluded)

Station 54 20 Years Wave Approach Angle (Degrees) = 120.0-149.9
Shoreline Angle* = 4.0 Degrees Azimuth
Water Depth = 10.00 Meters
Percent Occurrence (X1000) of Height and Period by Direction

Height (Meters)	Period (Seconds) 0.0-2.9	3.0-3.9	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-Longer	Total
0.00 - 0.49	10	.	.	.	.	.	.	.	.	.	10
0.50 - 0.99	.	.	.	.	.	.	.	.	.	.	0
1.00 - 1.49	.	.	.	.	.	.	.	.	.	.	0
1.50 - 1.99	.	.	.	.	.	.	.	.	.	.	0
2.00 - 2.49	.	.	.	.	.	.	.	.	.	.	0
2.50 - 2.99	.	.	.	.	.	.	.	.	.	.	0
3.00 - 3.49	.	.	.	.	.	.	.	.	.	.	0
3.50 - 3.99	.	.	.	.	.	.	.	.	.	.	0
4.00 - 4.49	.	.	.	.	.	.	.	.	.	.	0
4.50 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - Greater	.	.	.	.	.	.	.	.	.	.	0
Total	10	0	0	0	0	0	0	0	0	0	

Average HS(M) = 0.01 Largest HS(M) = 0.01 Angle Class % = 0.0

Station 54 20 Years Wave Approach Angle (Degrees) = 150.0-179.9
Shoreline Angle* = 4.0 Degrees Azimuth
Water Depth = 10.00 Meters
Percent Occurrence (X1000) of Height and Period by Direction

0.00 - 0.49	.	.	.	.	.	.	.	.	.	.	0
0.50 - 0.99	.	.	.	.	.	.	.	.	.	.	0
1.00 - 1.49	.	.	.	.	.	.	.	.	.	.	0
1.50 - 1.99	.	.	.	.	.	.	.	.	.	.	0
2.00 - 2.49	.	.	.	.	.	.	.	.	.	.	0
2.50 - 2.99	.	.	.	.	.	.	.	.	.	.	0
3.00 - 3.49	.	.	.	.	.	.	.	.	.	.	0
3.50 - 3.99	.	.	.	.	.	.	.	.	.	.	0
4.00 - 4.49	.	.	.	.	.	.	.	.	.	.	0
4.50 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - Greater	.	.	.	.	.	.	.	.	.	.	0
Total	0	0	0	0	0	0	0	0	0	0	

Average HS(M) = 0 Largest HS(M) = 0 Angle Class % = 0

(Sheet 3 of 3)

Table 7-2
Wave Conditions and Estimated Longshore Sediment Transport

Central Angle deg	Wave Height m	Period sec	Percent Occurrence	Breaking Wave Height, m	Breaking Wave Angle, deg	Longshore Transport Rate m^3/year
75	0.32	4.3	21.412	0.24	14.7	7,100
45	0.47	6.9	26.323	0.62	13.2	83,000
15	0.61	7.2	15.521	0.87	5.6	50,500
-15	0.61	6.9	21.167	0.86	-5.6	-67,100
-45	0.01	1.5	0.010	0.12	-18.1	-0

Gross Northerly Longshore Sediment Transport Rate: 140,600 m^3/year
Gross Southerly Longshore Sediment Transport Rate: -67,100 m^3/year
Net Longshore Sediment Transport Rate (North): 73,500 m^3/year

Table 7-3
Input Wave Data (10-m Depth), Breaking Wave Conditions, and Estimated Longshore Sediment Transport Rate (Adjusted Shoreline Angle) for Asbury Park to Manasquan Inlet, New Jersey

Central Angle deg	Wave Height m	Period sec	Percent Occurrence	Breaking Wave Height, m	Breaking Wave Angle, deg	Longshore Transport Rate, m^3/year
83	0.32	4.3	21.412	0.18	12.9	2,900
53	0.47	6.9	26.323	0.58	14.5	76,900
23	0.61	7.2	15.521	0.85	8.4	72,000
- 7	0.61	6.9	21.167	0.86	-2.7	-32,600
-37	0.01	1.5	0.010	0.12	-15.3	-0

Gross Northerly Longshore Sediment Transport Rate: 151,800 m^3/year
Gross Southerly Longshore Sediment Transport Rate: -32,600 m^3/year
Net Longshore Sediment Transport Rate (North): 119,200 m^3/year

7-3. West Coast Example: Oceanside, California

a. Site description. The Oceanside littoral cell is located along the southern California coast just north of San Diego (Figure 7-4). It is about 57 miles long, extending from Dana Point at the north end to Point La Jolla at the south end. Oceanside Harbor and its entrance jetties are located near the center of the cell and locally interrupt littoral processes. Notable beach erosion at Oceanside began when the first boat basin was constructed in 1942, and the erosion became severe after the harbor was expanded and the jetties were extended. A summary of the harbor development is given in Figure 7-5. Therefore, the shoreline change modeling effort in the sediment budget analysis focused on shorelines adjacent to the harbor and extending 4 miles north and south of the harbor, as discussed herein.

(1) Primary sources of sediment in the Oceanside littoral cell are rivers and beach nourishment. The long-term net direction of sand transport through the cell is believed to be from northwest to southeast. However, seasonality in transport direction is strong. Sand is typically transported to the north from May to October as the result of waves originating from southern hemisphere storms. At other times of the year, the transport is typically to the south. Dana Point, the northwest boundary of the cell, is a nearly complete littoral barrier (Everts, Bertolotti, and Anderson 1989). It extends

Table 7-4
Input Wave Data (10-m Depth), Breaking Wave Conditions, and Estimated Longshore Sediment Transport Rate (28 Wave Conditions), Asbury Park to Manesquan Inlet, New Jersey

Angle Band	Central Angle deg	Wave Height m	Period sec	Percent Occurrence	Breaking Wave Height, m	Breaking Wave Angle, deg	Longshore Transport Rate, m^3/year
1	83	0.25	4.1	17.072	0.14	12.0	1,300
.	.	0.75	5.1	4.128	0.39	17.0	5,200
.	.	1.25	6.0	0.201	0.64	20.1	1,000
.	.	1.75	6.6	0.011	1.87	22.6	100
2	53	0.25	7.2	18.059	0.35	11.1	11,900
.	.	0.75	6.1	5.511	1.81	17.9	45,300
.	.	1.25	6.7	1.918	1.28	21.8	58,400
.	.	1.75	7.5	0.628	1.75	24.7	46,100
.	.	2.25	7.7	0.178	2.18	27.4	24,300
.	.	2.75	8.9	0.026	2.99	31.3	8,500
.	.	3.25	9.5	0.003	3.08	31.8	1,100
3	23	0.25	7.5	8.843	0.42	5.9	4,900
.	.	0.75	6.5	4.108	0.98	9.3	29,700
.	.	1.25	6.6	1.404	1.50	11.3	35,800
.	.	1.75	7.3	0.562	1.92	12.4	28,900
.	.	2.25	7.9	0.425	2.57	14.0	50,500
.	.	2.75	8.2	0.150	3.06	15.1	29,600
.	.	3.25	9.5	0.021	3.59	16.1	6,500
.	.	3.75	9.5	0.008	4.05	17.0	3,500
4	-7	0.25	5.8	10.582	0.39	-1.9	-1,700
.	.	0.75	7.8	6.923	1.05	-2.8	18,500
.	.	1.25	8.1	2.313	1.56	-3.4	-19,800
.	.	1.75	8.4	0.919	2.16	-4.0	-20,800
.	.	2.25	8.3	0.340	2.66	-4.4	-14,200
.	.	2.75	9.0	0.083	3.18	-4.7	-5,900
.	.	3.25	10.8	0.005	3.72	-5.0	-600
.	.	3.75	10.8	0.002	3.96	-5.1	-300
5	-37	0.25	1.5	0.010	0.28	-22.9	-0

Gross Northerly Longshore Sediment Transport Rate: 392,600 m^3/year
Gross Southerly Longshore Sediment Transport Rate: -81,800 m^3/year
Net Longshore Sediment Transport Rate (North): 310,800 m^3/year

900 ft seaward from Dana Strand Beach and then continues submerged an additional 2,500 ft to depths of 40 to 60 ft. This rocky underwater protrusion is believed to permit only small quantities of sand to move around the headland. Scripps and La Jolla Submarine Canyons, located at the southeast end of the cell, are the ultimate repositories of sediment transported alongshore in the Oceanside littoral cell. There is no indication that sand bypasses these canyons and the Point La Jolla headland into the Mission Bay region. These canyons are important sediment sinks because they extend close to shore (Inman 1976). Point La Jolla has been considered a complete littoral barrier by a number of investigators (Shepard 1950; Inman 1953; Everts and Dill 1988).

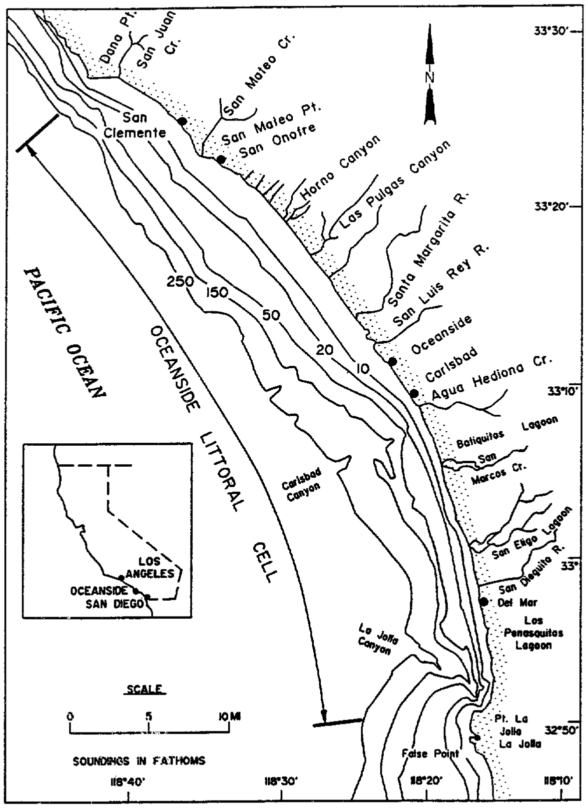

Figure 7-4. Oceanside littoral cell

(2) The Santa Margarita and San Luis Rey Rivers were once major sources of beach sediment for the Oceanside littoral cell. Neither has provided significant nourishment to the beaches in recent years, due in part to sand impoundment at flood control systems in their upper watersheds, but perhaps also due as much to a lack of extreme rainfall and subsequent large flood flows (Griggs 1987). Estimates of riverine sediment discharges into the coastal zone of the Oceanside littoral cell have been made by several investigators since the 1970s (Inman 1976; California Department of Navigation and Ocean Development (DNOD) 1977; Brownlie and Taylor 1981; Inman and Jenkins 1983; Simons, Li, & Assoc. 1988). The most detailed analysis of coarse sediment discharge to the ocean to date is that of Simons, Li, and Assoc. (1988); however, all river sediment discharge values are estimates. The range of estimates of sediment delivery to the coastal zone by the Santa Margarita and San Luis Rey Rivers is presented in Table 7-5.

b. Background. Six sets of aerial photographs were available to provide shoreline position measurements between March 1964 and January 1988. Historical shoreline positions were digitized at 100-ft intervals and were referenced to a baseline tied to the California state plane coordinate system. Data were analyzed over distances of 21,600 ft north and 21,600 ft south of Oceanside harbor to determine net change in shoreline position and average rates of change. During the period 1964 to 1974, the shoreline north of the harbor prograded an average 4.5 ft/yr. In the same period, the shoreline south of the harbor receded approximately 9.7 ft/yr, resulting in an average loss of 99.0 ft of beach. During this 10-year period, the only stretch of shore south of the harbor that exhibited progradation was between the south harbor jetty and the groin upcoast of the San Luis Rey River. This 10-year period selected was for examination because there are also wave hindcasts (WIS) available for the same period. These results were used to calibrate the shoreline change model.

(1) Erosion of the beaches south of the Oceanside Harbor complex and the accompanying accretion of sand in the entrance channel and harbor have been persistent problems since the construction of the Del Mar Boat Basin and the protective breakwaters in 1942-1943. Due to periodic sediment dredging and bypassing operations that transfer sand to Oceanside beaches, the harbor complex is a temporary sink in the middle of the littoral cell and traps sand moving in either direction (Figure 7-5). The northern breakwater acts as a partial barrier to southerly moving sand, trapping a portion of the sand on its north side until the shoreline realigns so that sand can move around the breakwater and into the entrance channel and harbor (Hales 1978). Sediment deposited in these areas is sheltered from wave action and littoral currents and cannot be transferred to the downdrift beaches except by mechanical means. Under conditions of northerly transport, the sand trapped north of the harbor tends to nourish the upcoast beaches (Hales 1978).

(2) Soon after construction of the north breakwater at the Del Mar Boat Basin in 1942, a large fillet of sand formed north of the harbor. Everts, Bertolotti, and Anderson (1989) observed that approximately 3,400,000 cu yd of sand accumulated north of the breakwater between 1942 and 1960. The shoreline advanced about 100 ft seaward along the reach from the breakwater to about 5.5 miles north of the structure. In the same period approximately 800,000 cu yd of material were

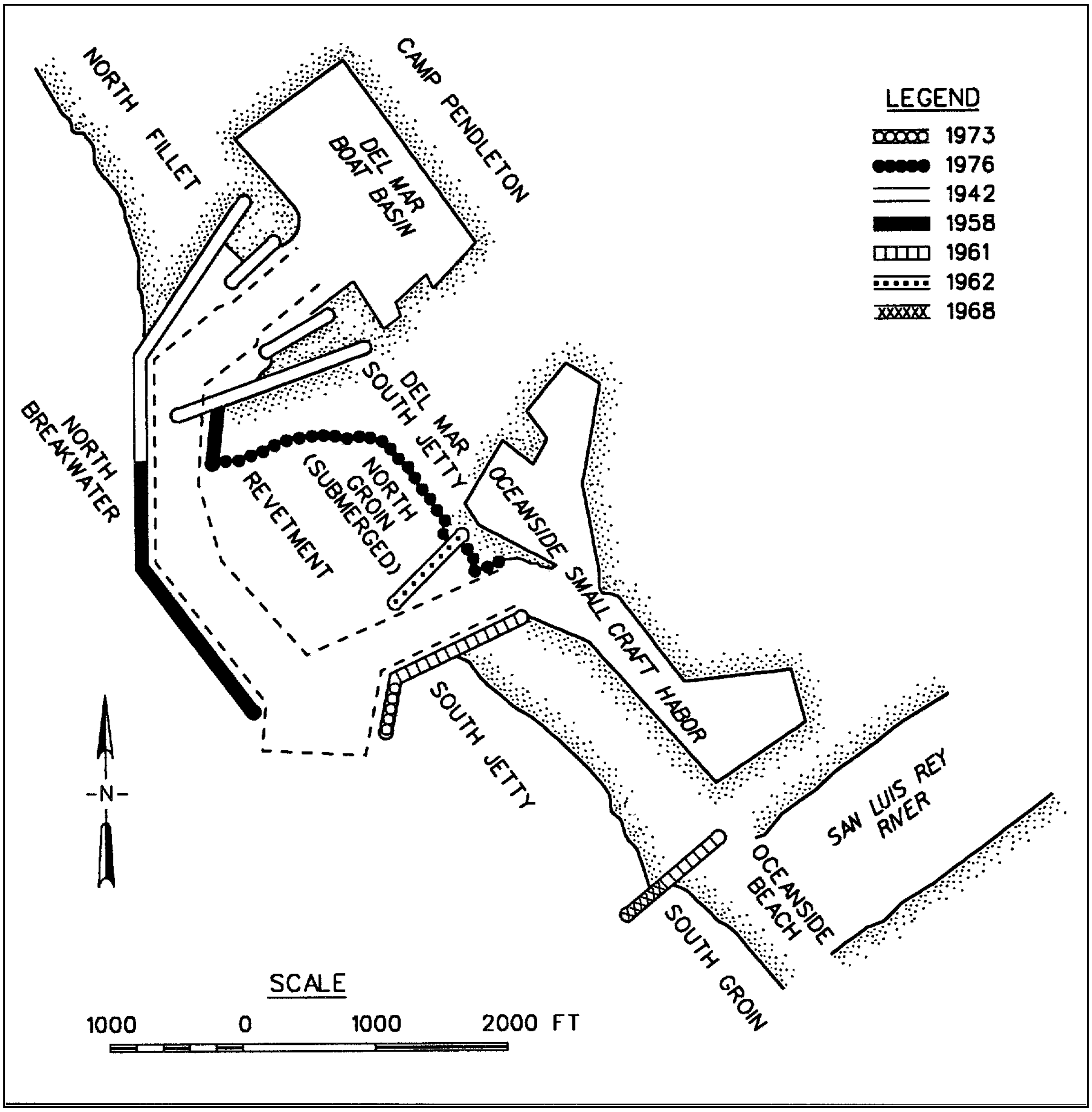

Figure 7-5. Del Mar Boat Basin and Oceanside Harbor

Table 7-5
Estimates of River Sediment Delivery, Oceanside, California, Littoral Cell

Investigator	Santa Margarita River cu yd/year	San Luis Rey River cu yd/year
DNOD (1977)	15,000	351,000
Brownlie and Taylor (1981)	11,300	12,500
Inman and Jenkins (1983)	24,000	37,000
Simons, Li and Assoc. (1988)	19,000	11,000

excavated from the harbor entrance channel and placed on Oceanside Beach. Also during this period, approximately 2,400,000 cu yd of sand were eroded from Oceanside Beach and the shoreface south of the harbor (in addition to the 800,000 cu yd of placed dredged material). Thus, a net 3,200,000 cu yd of sand was removed from Oceanside Beach. If all this material were lost after the breakwater was constructed, the rate of erosion at Oceanside Beach would be about 180,000 cu yd/year. The volume of sand accumulated and the rate of the fillet formation suggest the net longshore transport rate in the 1940s and 1950s averaged about 230,000 cu yd/year to the south (Everts, Bertolotti, and Anderson 1989).

(3) Since the mid-1960s, maintenance dredging of the entrance channel has been required on an almost annual basis. The average annual volume of material dredged from the entrance channel over the 17-year period between 1965 and 1982 is approximately 300,000 cu yd/year. Tekmarine, Inc. (1987) noted that two periods of distinctly different dredging rates appear to exist which constitute these average values. The dredging rate for the initial 6 years after the 1965 harbor expansion averaged 450,000 cu yd/year, but diminished by about one third to 293,000 cu yd/year for the succeeding 11 years. They believe the reason for such a substantial change in the dredging rate may be found in the change in disposal practices for beach nourishment operations beginning around 1971. As shown in Figure 7-6, until 1971 the disposal site was located relatively close to Oceanside Harbor, sometimes within 3,000 ft of the south jetty. The center of gravity of the disposed material was located about 7,000 ft from the south jetty. After 1971, the center of gravity of the disposed material was positioned about 11,000 ft from the south jetty.

c. Previous analyses. Three methods of estimating longshore sand transport rates for the Oceanside cell have been used in previous studies: fillet formation, beach erosion, and calculations of potential transport using either wave hindcasts or measurements and empirical predictive formulae. Marine Advisers (1961) developed a hindcast wave climate for Northern Hemisphere swell and local sea using weather maps from 1956-1958, and Southern Hemisphere swell using weather maps from 1948-1950. This data set was used to estimate potential longshore transport at Oceanside.

(1) Hales (1978) used a combination of Marine Advisers (1961) and California DNOD (1977) wave hindcast data. The DNOD (1977) statistics were considered quite reliable at that time. However, subsequent analysis has revealed their development suffered from computational limitations which may have introduced bias in the results. Hales (1978) calculated wave refraction to the break point and applied the Shore Protection Manual (1984) wave energy flux method to compute potential longshore transport. Island sheltering effects based on the work of Arthur (1951) were taken into consideration. Estimates using these hybrid deep water wave statistics produced an average annual transport to the south of 643,000 cu yd/year and a transport to the north of 541,000 cu yd/year for a net transport of 102,000 cu yd/year to the south.

(2) Inman and Jenkins (1983) produced the most complete estimate of potential longshore transport from hindcast data for this region. They also used a combination of Marine Advisers (1961) and DNOD (1977) wave data, but utilized DNOD Station 5 which is more energetic and farther away from the coast than a hypothetical Station 5-1/2 (halfway between Station 5 and Station 6) used by Hales (1978). This decision resulted in a stronger southerly transport than that obtained by Hales (1978). Inman and Jenkins (1983) estimates resulted in an average annual transport to the south of 807,000 cu yd/year and a transport to the north of 553,000 cu yd/year for a net transport of 254,000 cu yd/year to the south.

(3) Seymour and Castel (1985) computed potential longshore transport rates using wave parameters derived from data collected by seven nearshore pressure sensor arrays for the period between 1980 and 1982. That analysis showed the episodic nature of the transport rates, characterized by extreme variability in direction and volume on a day-to-day basis. Although the absolute values of the rates obtained appear to be too small, nevertheless, they statistically confirm that seasonal transport can be several times larger than the annual net transport. In the vicinity of Oceanside, half of the annual gross transport was calculated to occur during only 10 percent of the time. According to Seymour and Castel, because of the extreme variability, missing one day of observations could result in a reversal of the estimated direction of longshore transport for the entire year at Oceanside.

(4) Estimates of the potential longshore sand transport in the vicinity of Oceanside are summarized in Table 7-6. The first three listed works used essentially the same methodology and wave data base to estimate potential transport rates, hence they cannot be considered independent. The latter two estimates result from independent methods and the net rates are within the range determined by the first three methods.

(5) It is important to draw a distinction between the potential longshore transport and the actual longshore transport. Potential longshore sediment transport is an estimate of the maximum capacity of the breaking waves to carry sand alongshore in the presence of an unlimited supply of movable material. Conditions often exist which prevent the actual transport of sand from achieving the potential rate. Examples of such limitations include an absence of sediment supply (rocky headland or littoral barrier) and an armored but otherwise sandy beach. The longest cobble beach region in southern California is found from Oceanside to Carlsbad. Many of the cobbles armoring this particular

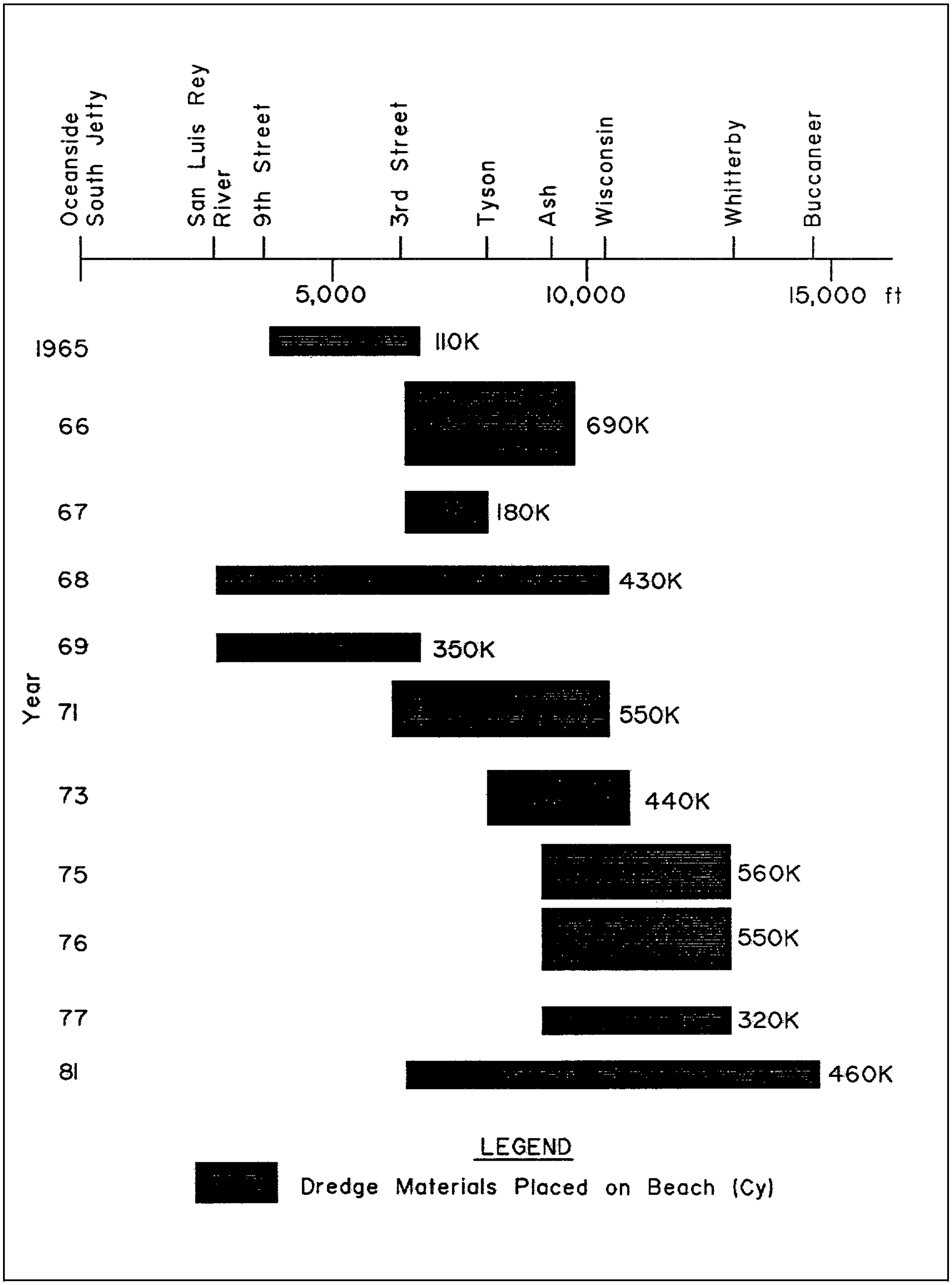

Figure 7-6. Beach nourishment placement locations (after Tekmarine 1987)

Table 7-6
Longshore Sand Transport Rate Estimates at Oceanside, California

Investigator	Method Used to Estimate Transport Rate	Transport Rate Estimates, cu yd/year Northerly	Southerly	Net
Marine Advisers (1961)	Potential transport equation	545,000	760,000	215,000
Hales (1978)	Potential transport equation	541,000	643,000	102,000
Inman and Jenkins (1983)	Potential transport equation	553,000	807,000	254,000
Everts et al. (1989)	Fillet formation in 1950s			230,000
Everts et al. (1989)	Beach erosion, 1942-1960			180,000

area were apparently dredged from the Oceanside Small Craft Harbor development in 1963 and placed on the beach. No method exists for estimating the longshore sand transport rate in the presence of cobble and sand mixtures in the littoral zone. It is known that a cobble beach will significantly reduce the longshore sand transport due to the armoring of sand particles.

(6) Cobbles along the beaches at Oceanside tend to stabilize the shoreline position. After nourishment, fill material is removed by waves and currents exposing the cobbles. The apparent beach erosion rate is significantly reduced. Seemingly, whatever volume of beach nourishment material is placed on the susceptible beach area is removed, and the cobbles are again exposed.

(7) The longshore transport rate should depend only on the magnitude of the longshore wave energy flux. The fact that it also appears to depend on the distance of travel between the disposal site and the harbor entrance channel indicated to Tekmarine, Inc. (1987) that the prevailing longshore processes were functioning at less than the potential of the wave climate. Armoring by cobble was believed to have contributed to this phenomena. Everts, Bertolotti, and Anderson (1989) noted that the mid- to late-1960s was a period of abnormally high southern hemisphere swell, and the northerly component of longshore energy flux may have been larger between 1965-1971 than between 1971-1982. This would account for the larger volume of material transported into the harbor during this time interval, and the longshore processes might still be functioning near their potential.

(8) Everts, Bertolotti, and Anderson (1989) developed two sediment budgets each for two control volumes in the vicinity of Oceanside. The first sediment budget was for the period 1942 through 1958, and the second was for the period 1958 through 1987. A north control volume extended about 30,000 ft upcoast from the north breakwater to Las Flores Creek. A south control volume extended about 18,000 ft downcoast from the south jetty at the harbor entrance to near Buena Vista Lagoon. Results of these budget analyses are listed in Table 7-7.

(9) Everts, Bertolotti, and Anderson (1989) found that in excess of 2,200,000 cu yd of sediment were deposited outside, but adjacent to, the harbor between 1942 and 1971. The deposit formed in response to the interruption of littoral processes at the north breakwater. The shoreline prograded as the end of the north breakwater was extended. By 1971 the new subaqueous deposit extended for the entire length of the north breakwater and along the south jetty of Oceanside Harbor, being broken only in its form by the dredged entrance channel. This trend toward natural bypassing around the harbor would have encompassed the entire harbor had the entrance channel not been periodically dredged. Everts, Bertolotti, and Anderson (1989) believe that significant quantities of sediment are no longer being withdrawn from the littoral system outside the harbor. They deduced that a critical fill volume of approximately 390,000 cu yd/year is required to maintain a dynamically stable shoreline at Oceanside (one that fluctuates seasonally about a steady mean position).

d. Wave conditions.

(1) Determination of the wave conditions is an essential step in the application of the shoreline change model. Deep water hindcast wave estimates from the WIS (Corson et al. 1986) were used to generate wave information in 65 ft of water near Oceanside.

Table 7-7
Sediment Budget, Oceanside, California (Everts, Bertolotti, and Anderson 1989)

Component	North Control Volume cu yd/year 1942-1958	1958-1987	South Control Volume cu yd/year 1942-1958	1958-1987
Santa Margarita River	+20,000[1]	+20,000		
Santa Margarita Delta	+25,000	0		
San Luis Rey River			+11,000	+11,000
San Luis Rey Delta			+25,000	0
Sea cliffs	+3,000	0	+1,000	+2,000
Coastal terraces	+28,000	+28,000	0	
Shoreface	+16,000	-54,000	+10,000	-33,000
Beach fills	0	0	0	+82,000
Sand mining	-14,000	0	0	0
Bypassing	0	0	+50,000	+355,000
Volume change with shoreline effects	+156,000	-32,000	-110,000	-50,000
Volume change seaward of north breakwater			+50,000	+55,000

Note:
1. + indicates volume gain; - indicates volume loss.

Two-dimensional energy spectra from a position seaward of the offshore islands were applied as the outer boundary condition for the nearshore wave model. Propagation of North Pacific wave spectra to the 65-ft-depth contour at WIS Station 7 (Figure 7-7) included the effects of island shadowing of wave energy. In addition, local wind effects were incorporated to estimate local seas (Jensen, Vincent, and Reinhard 1989). Only 41 percent of local seas represented events directed onshore (within ±90 deg of shore normal). All offshore-directed wave events were assigned zero energy for the shoreline change modeling.

(2) Southern hemisphere swell data were obtained from measurements made at the Olympics buoy from April 1984 to September 1985. Measurements of January-March and October-December were repeated to create a full 2-year time series. Limitations of the measurement interval necessitated repetition of this 2-year time series of Southern Hemisphere swell throughout the simulation period. The Southern Hemisphere swell wave information also was propagated to WIS Station 7.

(3) The separate wave data were synchronously combined to produce a single time series consisting of the three wave components. Seven angle bands were used to summarize the distribution of the spectral peak periods of the northern swell, the southern swell, and the local seas. Table 7-8 summarizes the distribution of wave energy spectral peak periods and angle bands of average directions at 6-hr intervals for WIS Station 7. These statistics include the three wave components for the period 1964-1974. This time period was selected, as previously mentioned, because both wave and historical shoreline position data were available.

(4) Northern swell is present for spectral peak periods between 5 and 20 sec, although the largest number of events occurs between 6 and 12 sec. In contrast, spectral peak periods for southern swell are typically 12 to 16 sec, with some as long as 20 sec. Directions of wave approach relative to the shoreline range from 55 deg north to 30 deg south of a line perpendicular to the coastline. Southern swell energy is more limited in direction, ranging only from 11 deg to 30 deg south of shore-normal.

(5) A statistical comparison of wave data calculated at 3- and 6-hr intervals for the 10-year period showed no significant difference in the distribution and magnitude of wave energy reaching the shoreline. In addition, the distribution of periods and angles was also nearly identical to that in the full 20-year time series. Thus, a time series of wave height, period, and angle at 6-hr intervals was selected as input for shoreline change modeling.

(6) To propagate the wave existing at the 65-ft contour onshore, it was necessary to develop a grid for the nearshore bathymetry. This information was obtained on magnetic media from the National Geophysical Data Center, Boulder, Colorado, and downloaded to disk for

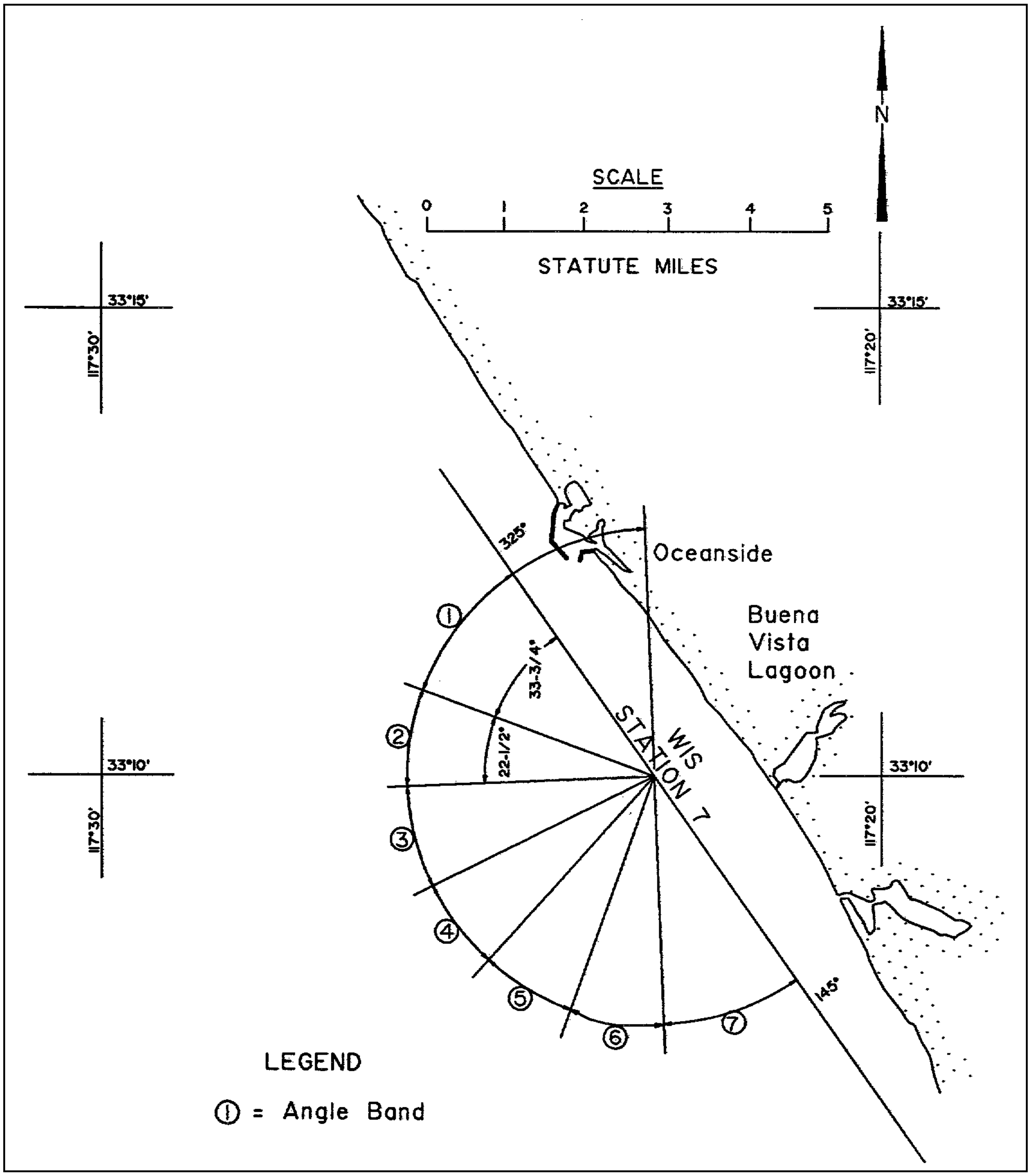

Figure 7-7. Orientation of wave angle bands at Oceanside

processing by a gridding software utility. A 3.2-mile by 10.3-mile rectangular grid was constructed for analysis of wave transformations. Over 9,800 data points were used to generate a 57- by 91-grid cell matrix of depths, with a 300-ft cross-shore dimension and a 600-ft long-shore dimension for each cell. The accuracy of this contour map was visually verified through comparison with National Oceanic and Atmospheric Administration (NOAA) Chart 18774 dated 1984.

(7) The wave transformation analysis over the grid of variable bathymetry was performed using the Regional Coastal Processes WAVE model (RCPWAVE) (Ebersole 1985; Ebersole, Cialone, and Prater 1986). The 46 wave period-wave angle conditions listed in Table 7-8 were transformed by RCPWAVE to approximately the 20-ft depth contour. Very little longshore variation in wave direction was present, a result of the relatively plane and parallel depth contours. The transformation

Table 7-8
Percent Occurrence of Waves by Period and Angle, Oceanside, California

Period	Angle Band						
s	1	2	3	4	5	6	7
5	7.69	7.22	0.87	0.22	0.27	0.27	0.10
6	0.01	1.24	0.28	0.09	0.31	-	-
7	-	5.81	2.79	0.23	0.32	-	-
8	-	2.37	2.53	0.25	0.11	-	-
9	-	1.69	2.39	0.39	0.18	-	-
10	-	2.48	1.96	0.24	0.50	-	-
11	-	4.87	2.53	0.21	0.88	-	-
12	-	4.99	5.52	0.87	3.28	-	-
14	-	0.56	8.89	4.63	9.34	-	-
16	-	0.01	1.95	5.67	2.22	-	-
20	-	-	0.02	0.81	-	-	-

grid covered a broader area than the shoreline change model to place lateral boundary effects far from the region of interest. Transformed wave height and angle information are stored at positions corresponding to the nominal 20-ft-depth contour. This analysis provides the nearshore wave information required by the shoreline change model. Transport rates were calculated for each of three wave components (southern swell, northern swell, and local seas) every 6 hr.

e. Description of shoreline change model.

(1) The numerical shoreline change model GENESIS (Hanson 1989; Hanson and Kraus 1989; Gravens, Kraus, and Hanson 1991) was used in this study. It simulates long-term evolution of beach plan shape and provides a framework to perform a time-dependent sediment budget analysis. The model is versatile in that it can describe a broad range of conditions encountered in shore protection projects. GENESIS has been adapted to the personal computer environment for use in planning on a local scale.

(2) GENESIS is formulated through a control volume approach, as discussed in paragraph 6-10. A change in the sand volume is produced by either a spatial gradient in the longshore sand transport rate and/or sources and sinks within the control volume. This change in volume represents either a seaward (accretion) or landward (erosion) displacement of the profile. The beach profile is assumed to have a constant shape.

(3) Longshore variation in sand transport is the major cause of long-term shoreline change on an open ocean coast. In GENESIS, the transport rate is calculated by Equations 6-22, 6-23, and 6-24.

(4) GENESIS is capable of simulating transport caused by multiple independent wave sources acting simultaneously. For example, in this study, northern swell, southern swell, and local seas are considered. At each 6-hr time step and at each model grid point alongshore, typically at 300-ft intervals, the volume transport rate $Q\ell$ in Equation 6-22 is determined as the vector sum of three independently calculated rates as

$$Q_{\ell} = Q_{NS} + Q_{SS} + Q_{LS} \tag{7-1}$$

in which Q_{NS} is the transport rate produced by the northern swell, Q_{SS} is the transport rate produced by the southern swell, and Q_{LS} is the transport rate produced by locally generated seas.

(5) GENESIS is capable of representing a wide range of natural processes and coastal engineering activities that influence shoreline change. The principal capabilities and limitations of the model are summarized in Table 7-9. Nearshore waves can be input directly, computed from offshore conditions using a wave transformation model such as RCPWAVE or calculated from offshore conditions using an internal subroutine in GENESIS if the offshore bathymetry is very regular. Information such as structure locations and configurations, beach fill locations and volumes, and river sediment discharge volumes must be entered. Measured shoreline positions are needed to calibrate and verify the model. The main outputs of GENESIS are longshore sand transport rates and the resulting shoreline change.

f. Boundary conditions.

(1) The proper specification of boundary conditions is essential for the successful implementation of a shoreline change model. If the boundary conditions are incorrect, the calculated results will be wrong. This is particularly true for long simulations.

(2) The Oceanside Harbor jetties interrupt the continuity of longshore processes in the vicinity of Oceanside. These jetties form boundaries which separate the study area into a north reach (the shoreline north of the harbor) and a south reach (the shoreline south of the harbor). Since accurate dredging records are available, this is a good position to locate a boundary.

Table 7-9
Capabilities and Limitations of GENESIS

Capabilities
Almost arbitrary numbers of groins, jetties, detached breakwaters, seawalls, beach fills, and river discharges
Structures and beach fills in almost any combination
Compound structures such as T-shaped groins and spur groins
Bypassing of sand around and transmission through groins and jetties
Diffraction at detached breakwaters, jetties, and groins
Wave transmission through detached breakwaters
Coverage of wide spatial extent
Offshore input waves of arbitrary height, period, and direction
Multiple wave trains (as from independent wave sources)
Sand transport produced by oblique wave incidence and by a longshore gradient in wave height
Highly automated, numerically stable, and well tested

Limitations
No wave reflection from structures
No tombolo development in a strict sense (shoreline not allowed to touch a detached breakwater)
Slight restrictions on location, shape, and orientation of structures
Basic limitations of shoreline change modeling theory

(3) The other ends of the two reaches are more difficult to specify. Plots of shoreline positions from six measurements between March 1964 and January 1988 showed that the north reach shoreline was relatively stable at a location about 4.1 miles north of the harbor. Shoreline data for the south reach exhibited similar trends about 4.1 miles south of the harbor. These locations were designated as the two remaining model boundaries. Since the observed shoreline moved only slightly at these locations, there must exist a very small (assumed zero) gradient in the longshore transport. This type of open boundary condition is referred to as a pinned-beach boundary in GENESIS. It allows sand to freely pass through the boundary.

(4) The idealized north reach is shown in Figure 7-8. Essential features are an open or pinned-beach boundary at the north end, the Santa Margarita River near the south boundary, and a diffracting jetty at the south boundary. The GENESIS grid for the north reach consists of 72 cells, each 300 ft long, for a total shoreline distance of 21,600 ft. The shoreline position at the northern end of the north reach was fixed at an average of the measured shoreline positions from available surveys.

(5) The Santa Margarita River intermittently discharges sediment to the north reach model area. Sediment discharge by southern California coastal streams is episodic and difficult to estimate for any one year. For the purposes of long-term shoreline modeling, average annual sediment discharge input each year during the storm season provides a reasonable approximation of the historical process. The average annual volume of sand and gravel for the Santa Margarita River, 19,000 cu yd (Simons, Li, & Assoc. 1988), was introduced uniformly at the shoreline in cells 9 through 14 every year at a constant rate from December 1 through March 31.

(6) The Oceanside north jetty is the only shore structure in the north reach. It was designated as a wave-diffracting source at the location of the change in jetty alinement, approximately 500 ft from shore, and was assigned zero permeability. No beach fills are known to have been placed along the north reach.

(7) The south reach incorporates the features shown in Figure 7-9. A diffracting jetty is located at the north boundary, a long groin (considered to be non-diffracting) was constructed in 1968 immediately to the north of the mouth of the San Luis Rey River, and

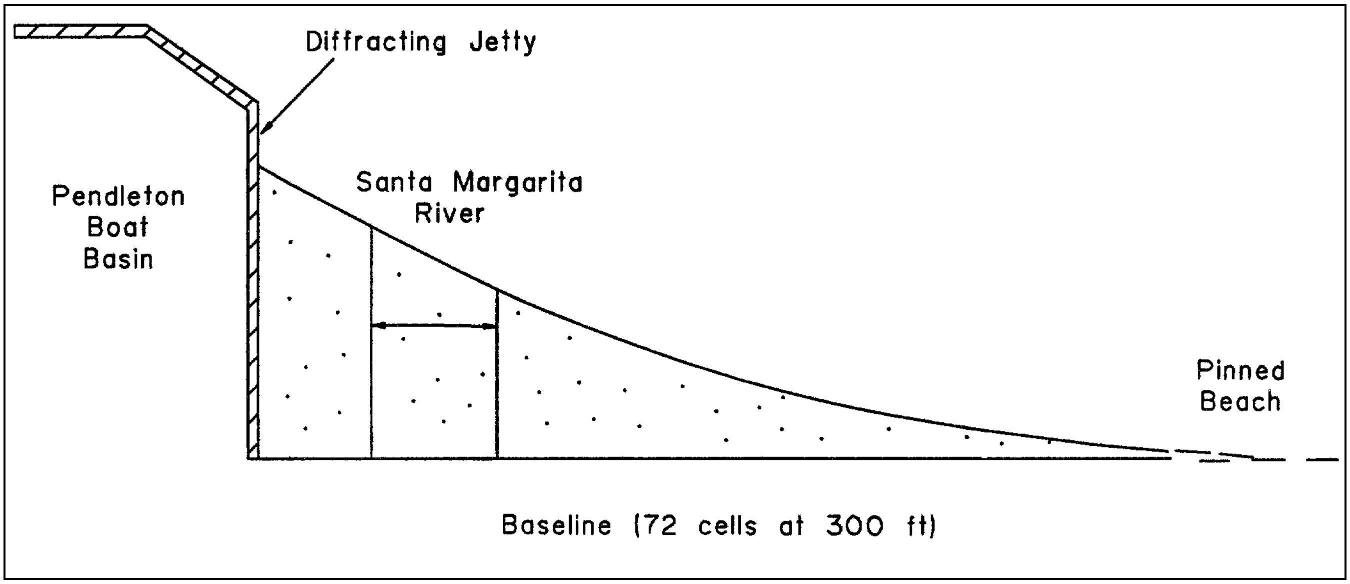

Figure 7-8. Idealized features of Oceanside north reach

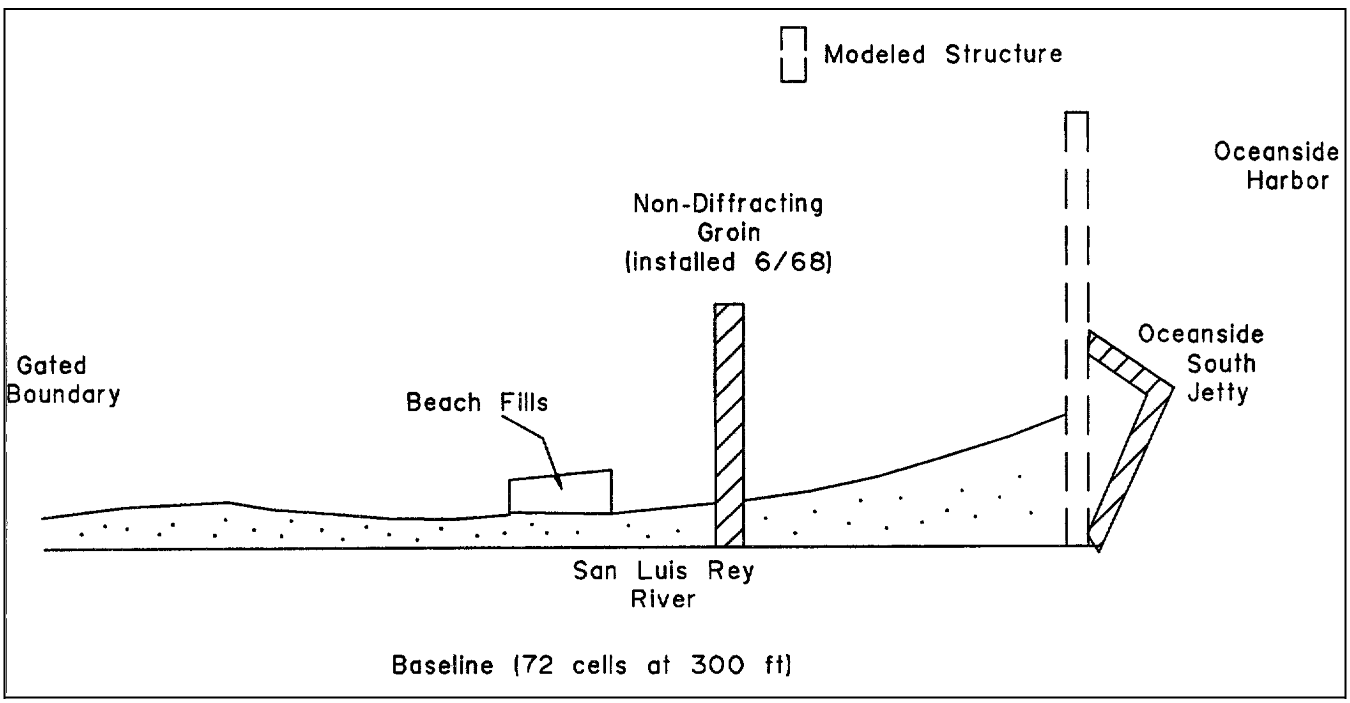

Figure 7-9. Idealized features of Oceanside south reach

several beach fills were placed to the south of the stream mouth. The south reach model grid also consists of 72 cells, each 300 ft long. At the south boundary, transport conditions were constrained to allow south-moving sand to move freely out of the study area, but to restrict north-moving sand onto the area (gated boundary). This represents the effects of the armored shoreline which existed during the calibration period south of the south reach (Hales 1978). Little sand was believed to be available for transport from Carlsbad to the shore at Oceanside.

(8) In the south reach, the San Luis Rey River input of sand and gravel was 11,000 cu yd annually at cells 63 through 65 from December 1 through March 31. This duration corresponds to the time in which winter storms are expected to cause the river to deliver sand to the coast.

(9) The south reach contains two structures. At the north boundary the end of the jetty was positioned to simulate the diffractive effects of the end of the outer breakwater at Oceanside Harbor. The groin at the San Luis Rey River mouth was incorporated in the model at the June 1968 simulation time-step. Zero permeability was assigned for both structures because the groin and south jetty have been grouted to prevent northerly sand movement through them. Shoreline surveys in the vicinity of the Oceanside pier indicate the pier has an insignificant effect on shoreline configuration, so it was not included in the model. Its location is shown on figures with model results for general orientation.

(10) A revetment was constructed in the area encompassed by the south reach prior to 1964 to harden the shoreline for upland protection. The revetment was implemented in the model as a seawall from cells 39 through 60 (extending approximately from 9th Street to Wisconsin Avenue).

(11) The shoreline in the south reach is an erosional shoreline and seven documented beach fills were placed in the reach during the calibration period. Initial placement volumes, alongshore extent of placement, and calculated berm widths are listed in Table 7-10. Experience with placing dredged material along this coast suggests modeling the placed volume by reducing the initial quantity by 20 percent to account for loss of fine material, and further reducing the remaining volume by 15 percent to account for losses from the system during profile adjustment. These volume adjustments were made before calculating berm widths listed in Table 7-10, which were then input to GENESIS.

Table 7-10
Beach Nourishment, Oceanside, California, 1965-1973

Date	Volume cu yd	Length ft	Effective Added Berm Width ft
1965	111,000	3000	16
1966	684,000	3600	79
1967	178,000	2100	35
1968	434,000	8100	22
1969	353,000	4200	35
1971	552,000	4200	55
1973	434,000	3000	45

(12) At Oceanside beach, the sea cliffs are protected by structures and the sediment yield from bluff erosion is negligible (Everts, Bertolotti, and Anderson 1989).

g. Beach profile

(1) Repetitive surveys were made at 12 transects in the Oceanside littoral cell between 1983 and 1988 as part of the Coast of California Storm and Tidal Wave Study (CCSTWS) field data collection program. Nine transects located in the study area were surveyed from three to eight times between 1983 and 1988. Profile characteristics were examined to evaluate parameters required in the shoreline change simulation.

(2) The closure depth defines the seaward limit of effective profile change. It is estimated by determining the depth at which significant profile changes cease to occur. To estimate closure depth, the standard deviation of depth was calculated as a function of mean depth at specified positions along each transect, following the procedure of Kraus and Harikai (1983). The depth of closure was determined to be the depth at which the variation in standard deviation decreased to a relatively constant amount and was estimated to be 30 ft in the Oceanside area. The berm height was determined from plotted profiles to be 14 ft relative to mean lower low water (MLLW). The zone of profile change extends from the berm crest to the closure depth, a total of 44 ft for the Oceanside model reaches.

(3) Another profile-related parameter required in GENESIS is the shape factor associated with the idealized equilibrium profile. The shape factor is calculated in GENESIS for a typical median grain diameter using an empirical formulation provided by Moore (1982). The effective grain size was obtained by comparing equilibrium profiles with mean nearshore profiles. The mean profile was calculated as the average of all surveys made at a transect. Equilibrium

profile curves generated for a range of sand sizes were compared with mean profiles. An effective median grain diameter of 0.28 mm was selected for input to GENESIS based on that comparison. Most sediment samples taken in water depths less than 15 ft in the study area from 1983 to 1988 had median grain sizes ranging from 0.16 to 0.50 mm. The input grain diameter determined from profile shape was, therefore, approximately the size determined statistically from samples.

h. Model calibration.

(1) The general calibration procedure for GENESIS requires determination of the longshore transport calibration parameters K_1 and K_2 in Equations 6-23 and 6-24 by reproducing measured shoreline changes that occurred in the study area. After initial model setup, calibration simulations were made in which the transport parameters and the passage of sand at the south boundary of the south reach model were varied. Computed shoreline change and longshore transport rates were optimized with $K_1 = 0.3$ and $K_2 = 0.2$.

(2) Simulated shoreline change for the calibration period is plotted for the north reach in Figure 7-10. Comparison of measured and calculated shoreline change near the harbor jetty shows reasonable agreement. The measured shoreline showed an advance over the entire reach length. The bulge in the middle of the reach could not be reproduced, degrading the quality of the simulation. The calculated average change in shoreline position was 2.5 ft/year advance, compared with a measured average of 4.5 ft/year advance.

(3) Only small variations in transport exist along the north reach because of the nearly plane and parallel offshore depth contours. The average net longshore transport rate was 430,000 cu yd/year to the south. At the south boundary of the north reach, the rate decreased to approximately 370,000 cu yd/year. This resulted in a shoreline advance and fillet formation near the north harbor jetty. Mean northerly and southerly sand transport rates averaged 100,000 cu yd/year and 530,000 cu yd/year, respectively. The magnitudes of the individual northerly and southerly sand transport rates differ somewhat from estimates presented in Table 7-6, but the net and the direction of net transport are consistent with previous estimates.

(4) The 1974 calculated south reach shoreline is plotted in Figure 7-11, along with the initial 1964 and the measured 1974 shorelines. Average simulated shoreline recession was 5.9 ft/year as compared with a measured rate of 9.7 ft/year. Comparison of measured and calculated shoreline change trends again indicates good agreement. The largest deviations from measured trends in shoreline response occurred midway along the reach where a very large beach fill (3.8 million cu yd) was placed in 1963, one year before the date of the initial calibration shoreline. Profile adjustment was probably continuing during the early part of the calibration period and may have contributed to greater shoreline recession rates than those calculated by GENESIS.

(5) The calculated average annual transport rates were 100,000 cu yd/year to the north and 360,000 cu yd/year to the south for a net transport of 260,000 cu yd/year to the south. Greater impact of wave variability exists along the south reach where the mean net transport is relatively low and reverses from northerly at a position north of the San Luis Rey River mouth to southerly immediately south of the river mouth. The southerly transport increases with increasing distance south of the river mouth to a point about 10,000 ft south of the harbor. This spatial change in transport results from diffraction and sheltering of waves by the Oceanside breakwater. Along the southern 10,000 ft of the model reach, calculated net sand transport is uniformly 400,000 cu yd/year to the south, approximately equal to the net rate in the north reach.

(6) The calculated transport is predominantly to the south but at a rate somewhat greater than previously estimated (see Table 7-6). Although the direction of transport is consistent with earlier estimates, differences in magnitude may be related to the use of a complete time series of wave data in the present study rather than statistical wave summaries as done in previous studies. In addition, the method used to obtain breaking wave parameters for the transport calculations should provide more accurate results because wave transformation is modeled using the actual bathymetry and local shoreline orientation. Finally, by matching calculated and measured shorelines, net transport rates are potentially more realistic than estimates obtained without these constraints.

i. Sediment budget.

(1) Quantitative information on beach fill volumes, river discharges, and shoreline losses and gains were combined with calculated longshore sand transport rates to produce a sediment budget for the period 1964-1974 (Figure 7-12). The present analysis represents an optimal agreement between measured and calculated

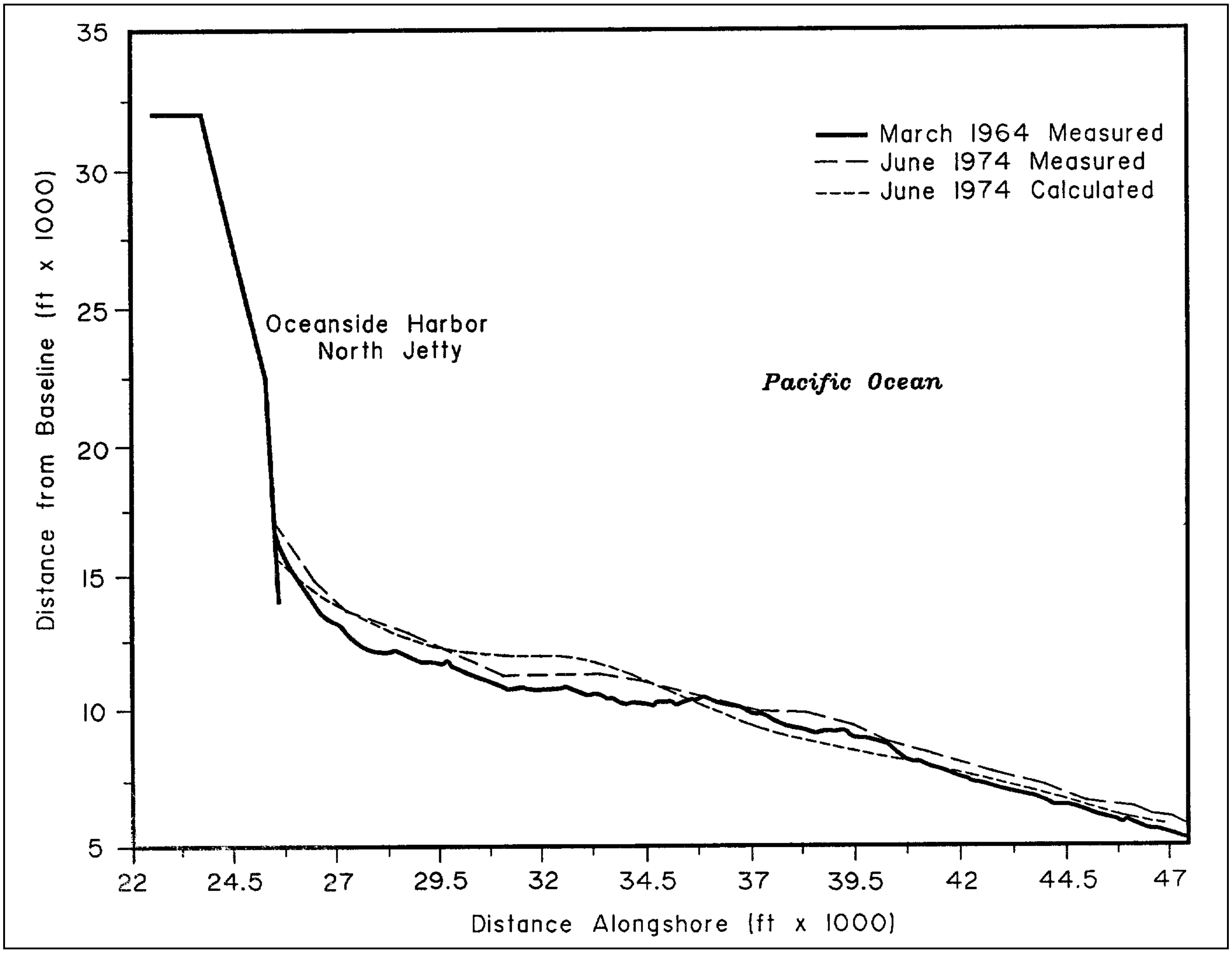

Figure 7-10. Oceanside north reach, 1964-1974

shoreline position and imposition of realistic boundary conditions. Volumes passing the lateral and shoreward sides of the control volume are known, whereas volumes passing the seaward boundary are derived by balancing the sediment budget. Estimated sediment exchange with the offshore was compared with amounts presented in other studies (Everts, Bertolotti, and Anderson 1989). For example, Weggel and Clark (1983) estimated that the amount of sediment lost to the offshore at the harbor ranged between 249,000 and 263,000 cu yd/year. Inman and Jenkins (1983) reported that about 48,000 cu yd/year were deflected offshore at the north jetty after it was extended in 1958.

(2) For the north reach, sand moves to the north out of the study area at an average rate of 90,000 cu yd/year and enters from the north at a rate of 540,000 cu yd/yr. In addition, the Santa Margarita River adds 19,000 cu yd/year to the budget, and shoreline accretion removes 99,000 cu yd/year. Although an estimated 210,000 cu yd/year of sand exits the north reach in a southerly direction, it does not bypass Oceanside Harbor but instead is deposited in the entrance channel and harbor complex. Some portion of this sand on the bypass shoal will enter the harbor during periods or seasons of southerly waves, and appear to be sediment that has arrived from the south. Of the 210,000 cu yd/year estimated to arrive from the north reach, 50,000 cu yd/year was placed conceptually to arrive from the south reach, corresponding to the ratio of average annual transport rates to the north and to the south described in paragraph *h* (5). The volume estimated to deposit in the channel and harbor is the average annual volume of beach fill, less the fines fraction, that was placed along

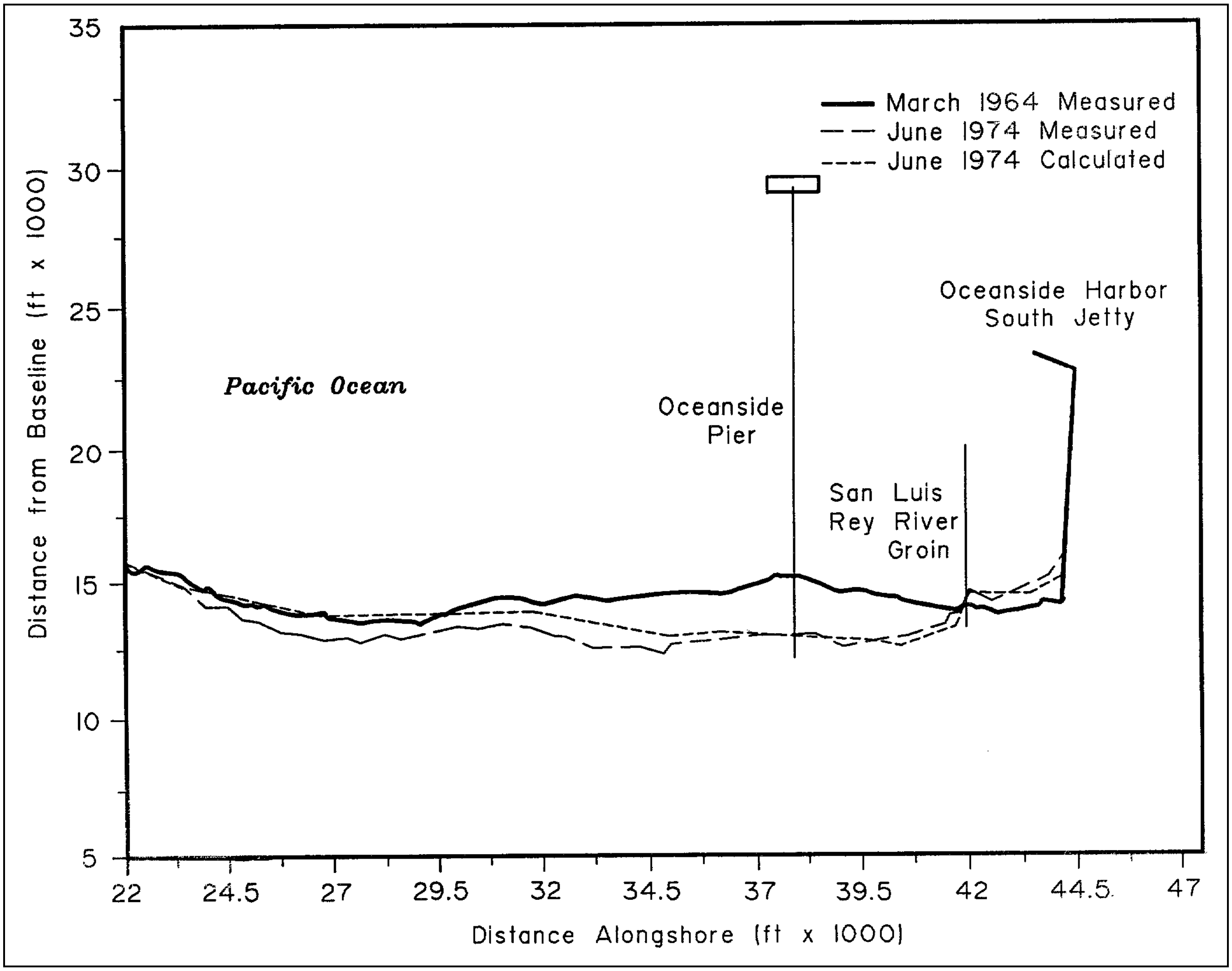

Figure 7-11. Oceanside south reach, 1964-7974

the south reach during this time interval. In closing the sediment budget to obtain the volume crossing the seaward boundary, approximately 160,000 cu yd/year were found to be lost from the system. It has been speculated that this offshore transport may be the source of sediment accretion observed in water depths up to 60 ft (Dolan et al. 1987). Because the shoreline change model uses a profile of fixed slope, a decrease in actual beach slope near the north jetty through sand impoundment is not represented. The associated portion of material accumulated inside the control volume would not appear in the budget and, therefore, would not be included in the seaward loss to balance the budget.

(3) Because of the recently completed grouting operations, the Oceanside Harbor complex is believed to now completely block sediment movement in either direction at the north boundary of the south reach. If substantial sand were to move north and past the San Luis Rey River groin, it would be expected that the shoreline would advance on the south side of the groin. This has not been the case in the past several years. At the south boundary, 400,000 cu yd/year of sand are transported out of the reach, and sand transport to the north is nearly zero. The San Luis Rey River delivers about 11,000 cu yd/year of sand and gravel to the south reach (Simons, Li, & Assoc. 1988), and shoreline loss accounts for 209,000 cu yd/year. Historical records show that the rate of beach nourishment of the south reach was 265,000 cu yd/year during the simulation interval. It is assumed this volume came from the Oceanside harbor and channel and that all material

dredged from the harbor was placed as fill in the south reach. Approximately 30,000 cu yd/year of beach fill material were transported to the offshore during profile adjustment whereas 20 percent of the initial beach fill, or roughly 55,000 cu yd/year, were considered material finer than beach sand and removed from the south reach to the offshore. This fine sediment may be derived from the continuously suspended material in the surf one or the soil type into which the harbor was dredged.

(4) It is emphasized that the sediment budget shown in Figure 7-12 and similar figures represents a best estimate of average annual rates and trends over a long time period. It may not provide a good estimate of sediment transport for any one particular year. The significance of temporal variations in longshore sand transport on development of a sediment budget can be examined by noting the range in values calculated at the boundaries of the reaches. At the north boundary of the north reach, calculated annual southerly directed sand transport ranges from 333,000 cu yd to 840,000 cu yd. The standard deviation is ±160,000 cu yd or 30 percent of the average annual rate. Calculated annual transport of sand exiting the reach to the north ranges from 66,000 cu yd to 111,000 cu yd, with a standard deviation of ±22,000 cu yd/year. Greatest variability is associated with sand passing the south boundary of the south reach where the standard deviation (±140,000 cu yd/year) represents 35 percent of the average yearly transport rate.

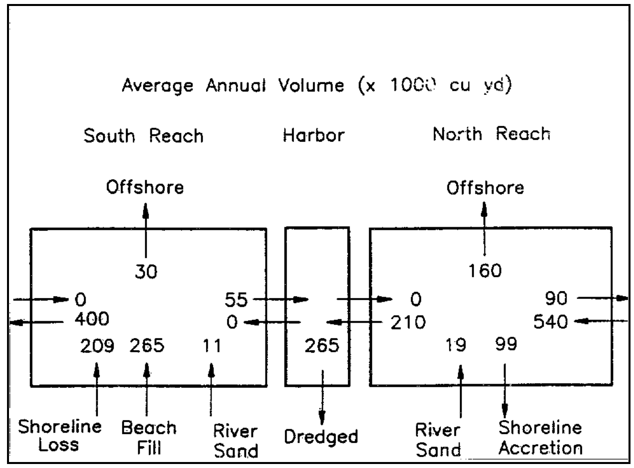

Figure 7-12. Sediment budget for Oceanside